b) K_0 – consolidation condition.

In field → K_0 – state.

In lab → Generally isotropic state (triaxial test) to simplify the field conditions.

< For sedimented soils>

* Field : During sedimentation and subsequent loading (no lateral strain), soil structure is formed to resist efficiently to vertical loading.

  (→ anisotropy)

∗ Lab : During consolidating for undisturbed anisotropic samples isotropically, soil structure can be altered to have isotropic-inclined characteristics. And total confining stress $p' = (\sigma_1' + \sigma_2' + \sigma_3')/3$ is different from (larger than) that in field.

- The effect of **isotropic consolidation** on undrained behavior of N.C. or lightly O.C. clays. ($K_0 < 1 \rightarrow p'_{iso} > p'_{Ko}$ and rearrange of soil structure during isotropic consolidation (more isotropic structure))

  ⇒ Comparing the shear behaviors under isotropic consolidation with those under K_0-consolidation

  ⇒ Subsequent shearing also induces structure rearrangement to activate the effective resistance to the given shearing mode.

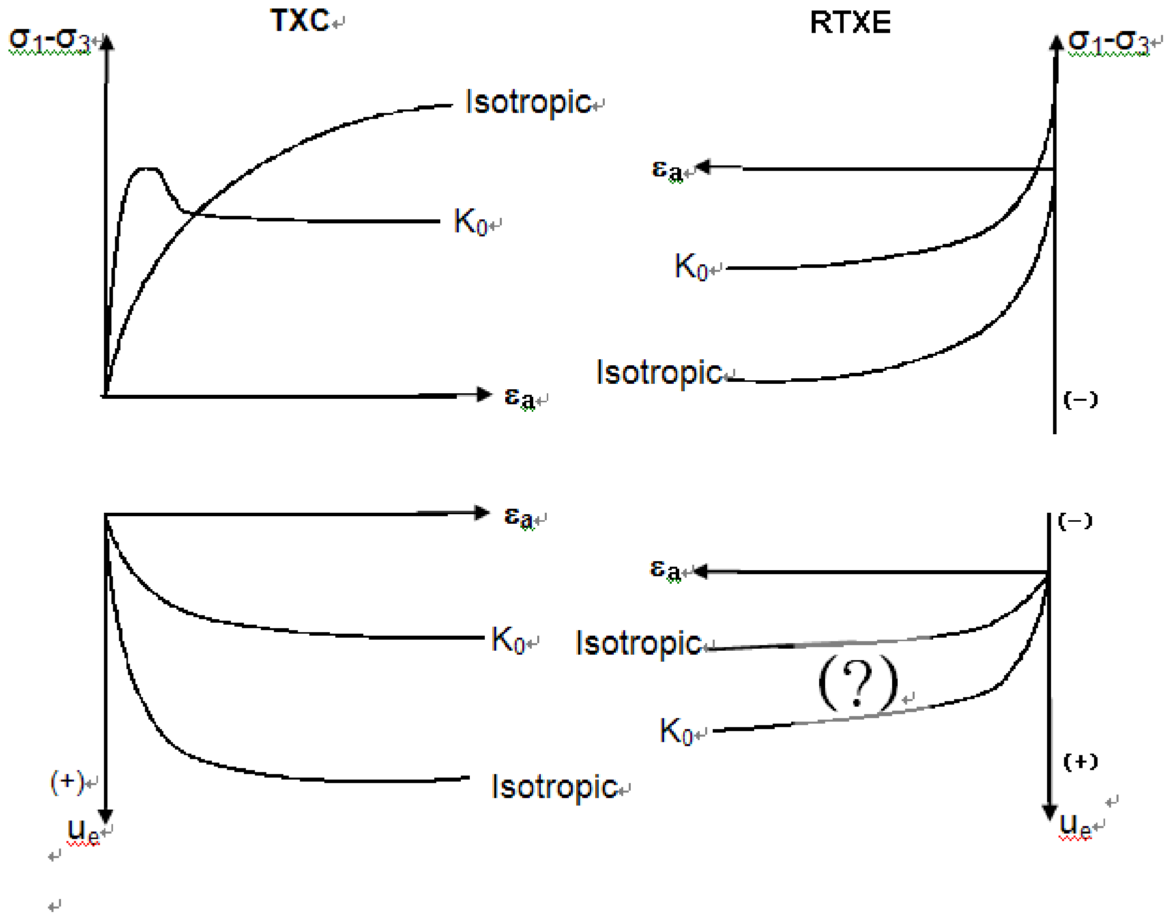
(1) Compression shearing.

- Soil structure: needs more strain to the peak and develops higher excess pore pressure and lower s_u.
- Higher p' induces higher s_u and possibly higher excess pore pressure during shearing.

(2) Extension shearing.

- Soil structure: increases s_u, and possibly (but not significantly) less strains to the peak and lower excess pore pressure.
- Higher p' induces higher s_u and higher excess pore pressure.

(3) Isotropic consolidation has no effect on ϕ' .


TXC
RTXE
σ1-σ3
Isotropic
K0
εa
(−)
(+)
ue
(?)

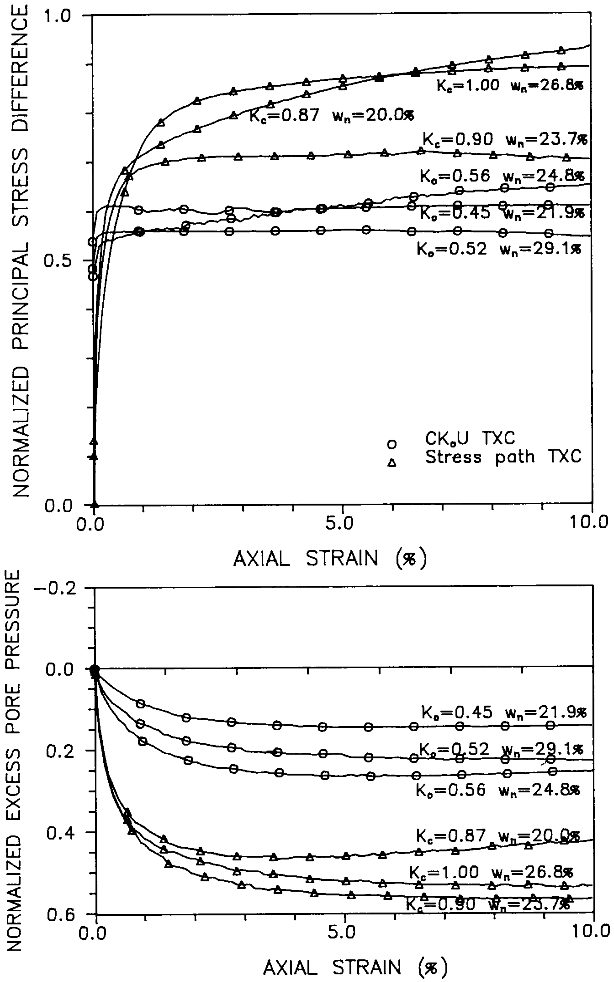


Figure 4-14 Consolidation Stress Ratio Effects on Stress-strain and Pore Pressure-Strain Response for Normally Consolidated Clays: Compression

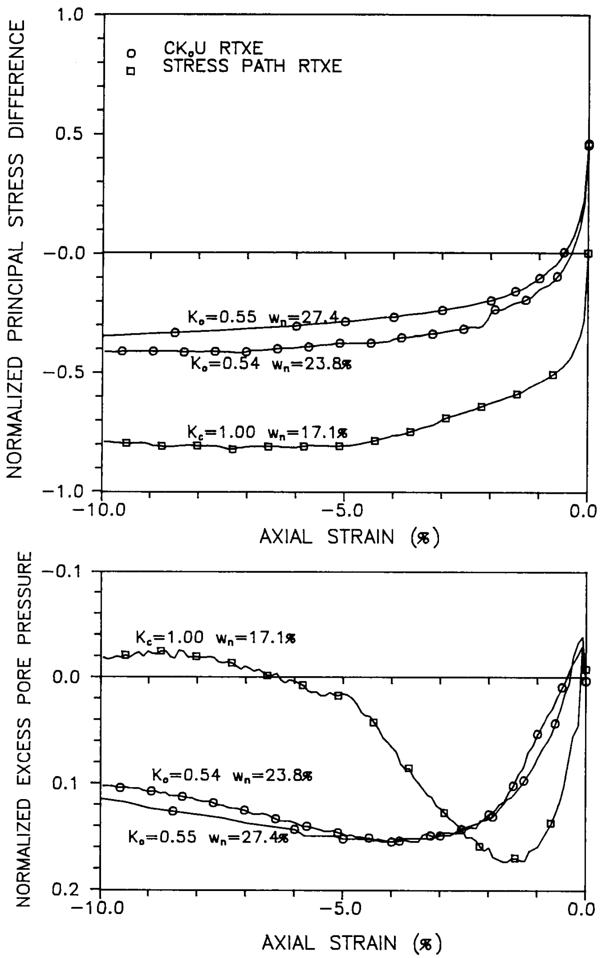


Figure 4-15 Consolidation Stress Ratio Effects on Stress-strain and Pore Pressure-Strain Response for Normally Consolidated Clays: Extension

∗ The Effect of Isotropic Consolidation on Shearing Behavior.

	Triaxial Compression	Trialxial Extension
Soil Structure Change	s_u ↓ (Stiffness ↓) (u_e ↑)	s_u ↑ (Stiffness ↑) (u_e ↓)
Increasing p’	s_u ↑ u_e (↑)	s_u ↑ u_e (↑)

∗ Anisotropy of s_u (for NC or lightly OC clays).

$$\left(\frac{(s_u)_E}{(s_u)_C}\right)_i > \left(\frac{(s_u)_E}{(s_u)_C}\right)_{Ko}$$

∗ Mayne (1985), for 42 soil types,

For comp. , $\left(s_u / \sigma'_{vc}\right)_{Ko} \approx 0.87\left(s_u / \sigma'_{vc}\right)_{iso}$

For ext. , $\left(s_u / \sigma'_{vc}\right)_{Ko} \approx 0.60\left(s_u / \sigma'_{vc}\right)_{iso}$

∗ Sivakugan and et al.

Base on using K_0 (=1-sinφ') and pore pressure parameter at failure, A_f for isotropic and K_0 – consolidation;

$$\frac{\left(\dfrac{s_u}{\sigma'_{vc}}\right)_{CKoUC}}{\left(\dfrac{s_u}{\sigma'_{vc}}\right)_{CIUC}} = \frac{K_0 + 2(1-K_0)A_{f,i}}{K_0 + 2(1-K_0)A_{f,Ko}}\left(A_{f,i}(1-K_0) + K_0\right)$$

∗ Wroth

$$\frac{\left(\frac{s_u}{\sigma'_{vc}}\right)_{CKoUC}}{\left(\frac{s_u}{\sigma'_{vc}}\right)_{CIUC}} = \frac{3-2\sin\phi'}{3}\left(1-a^2\right)^{\Lambda}$$

where $a = \frac{3-\sin\phi'}{2(3-2\sin\phi')}$, $\Lambda = 1-\frac{C_r}{C_c}$

∗ For heavily OC clays

$$K_0 = (1-\sin\phi)(OCR)^{\sin\phi} \Rightarrow For\ \ OCR = 4,\ K_0 \approx 1$$

So, the higher OCR (<4), the lower anisotropy.

2) <u>Shearing Conditions</u>.

a) Anisotropy

2 types
- 1) Material → Inherent anisotropy.
- 2) $K_0 \neq 1$ → Stress system anisotropy .

→ Caused by tendency of clay particles to become horizontally oriented during deposition (1 – D Compression).

Anisotropy will affect

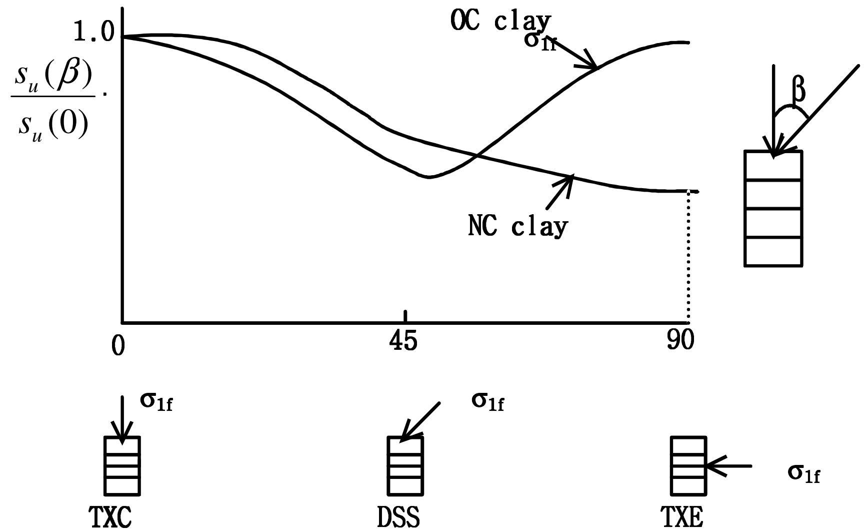
→ ϕ' , c' ⇒ $\phi'_{ext} > \phi'_{com}$ (?).

→ s_u.

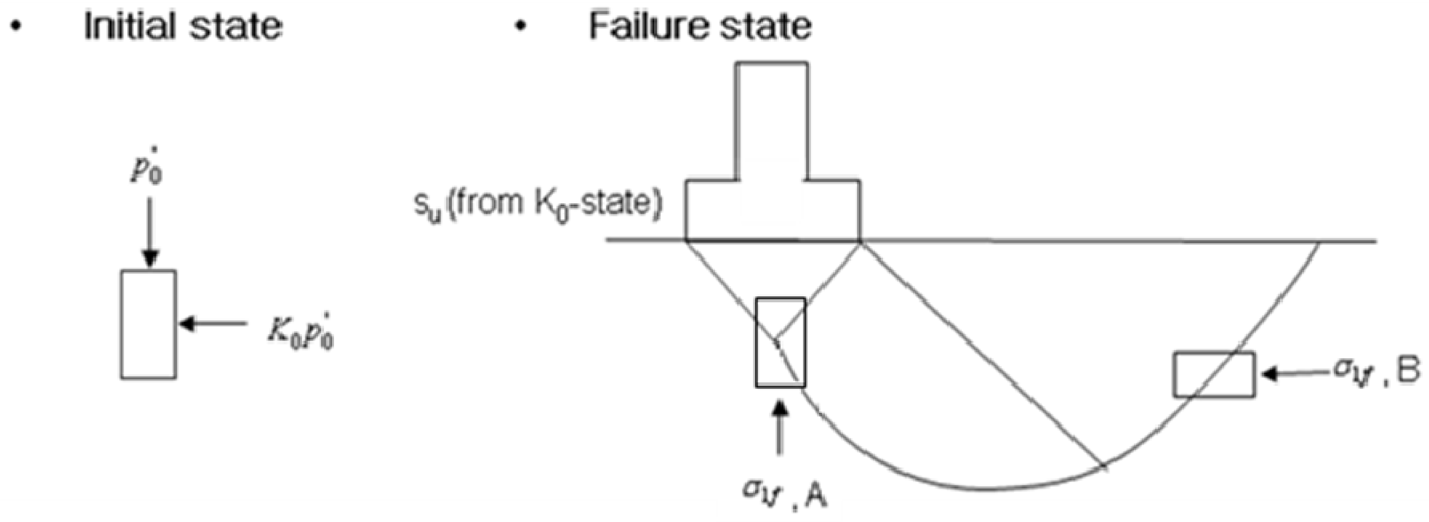
→ Deformation parameters (σ - ε response).

→ Pore pressure response.

As the direction of major principal stress changes, anisotropy of undrained strength ($s_u(\beta)/s_u(0)$) is shown as,

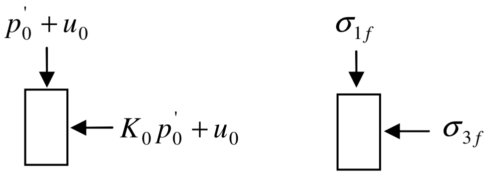


- Stress reorientation effects

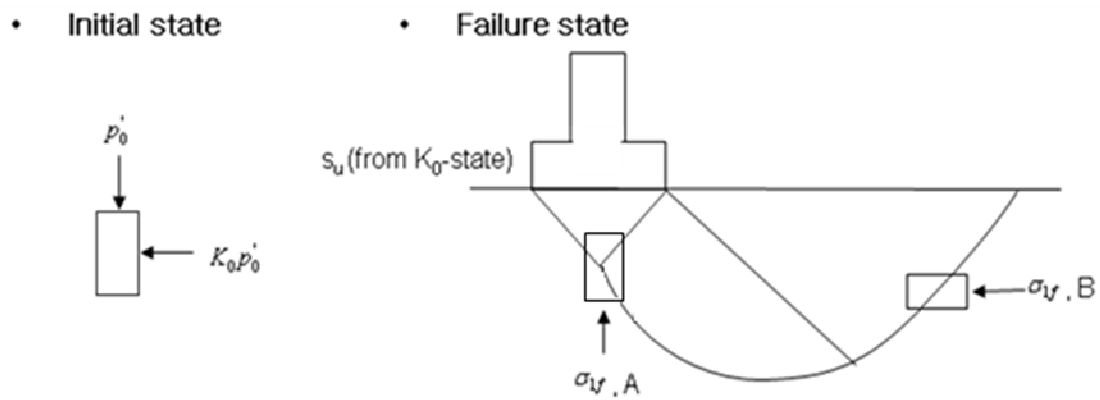


We want to evaluate the change in strength caused by the rotation of principal stress direction (⇐ stress system induced anisotropy due to $K_0 \neq 1$).

Point A



Init. Fail.



$$s_u = \frac{\sigma_{1f} - \sigma_{3f}}{2} = \frac{(\sigma_1 - \sigma_3)_f}{2}$$

Look at N.C. clays (c'=0), based on Mohr-coulomb criteria to define failure,

$$\sin\phi^{'} = \frac{(\sigma_1 - \sigma_3)_f}{\sigma_{1f}^{'} + \sigma_{3f}^{'}}$$

By Algebra,

$$(\sigma_1 - \sigma_3)_f = \sigma'_{3f}\left(\frac{2\sin\phi'}{1-\sin\phi'}\right) \text{ ------------------------------------ ①}$$

Where $\sigma'_{3f} = \sigma_{3f} - \Delta u - u_0$

$$= \sigma'_{3i} + u_0 + \Delta\sigma_3 - \Delta u - u_0 = \sigma'_{3i} + \Delta\sigma_3 - \Delta u$$

We know : $\Delta\sigma_1 = \sigma_{1f} - p'_0 - u_0$

$$\Delta\sigma_3 = \sigma_{3f} - K_0 p'_0 - u_0$$

$$\Delta u = B(\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3))$$ (for saturated soil, B=1)

$$\begin{aligned}\sigma'_{3f} &= \sigma'_{3i} + \Delta\sigma_3 - \Delta u \\ &= \cancel{K_0 p'_0} + (\cancel{\sigma_{3f}} - \cancel{K_0 p'_0} - \cancel{u_0}) \\ &\quad -[\cancel{\sigma_{3f}} - K_0 p'_0 - \cancel{u_o} + A(\sigma_{1f} - p'_0 - \cancel{u_0} - \sigma_{3f} + K_0 p'_0 + \cancel{u_0})]\end{aligned}$$

$$\sigma'_{3f} = K_0 p'_0 - A(\sigma_{1f} - \sigma_{3f} - p'_0 + K_0 p'_0) \text{ ------------------------ ②}$$

Sub. ② → ①.

$$(\sigma_1-\sigma_3)_f = [K_0 p_0' - A((\sigma_1-\sigma_3)_f - p_0' + K_0 p_0')](\frac{2\sin\phi'}{1-\sin\phi'})$$

$$(\sigma_1-\sigma_3)_f(1+\frac{2A\sin\phi'}{1-\sin\phi'}) = [K_0 p_0' - A(-p_0' + K_0 p_0')](\frac{2\sin\phi'}{1-\sin\phi'})$$

$$\Rightarrow \frac{(\sigma_1-\sigma_3)_f}{2p_0'}\left(=\frac{s_u}{p_0'}\right) = [K_0 - A(K_0-1)](\frac{\sin\phi'}{1-\sin\phi' + 2A\sin\phi'})$$

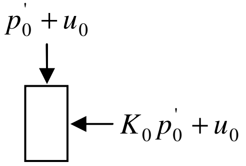
<u>NC clays</u> (typical values)

For ϕ'=30°

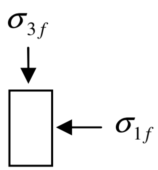
K_0 = 0.5

A_f=1 $\Rightarrow \frac{s_u}{p_0'} = 0.3$

Point B

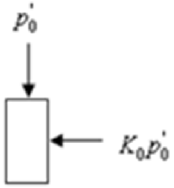


Init.

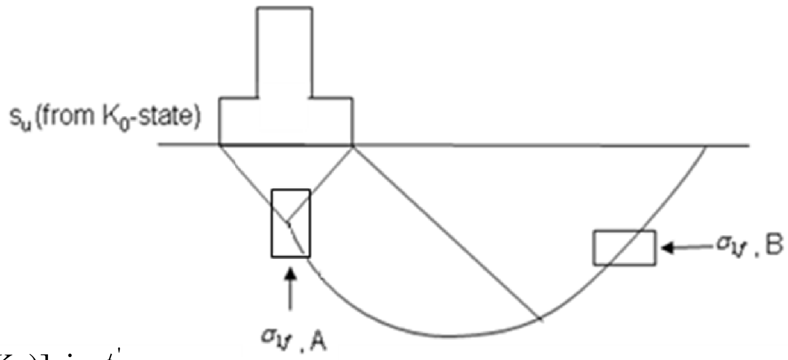


Fail.

- Initial state



- Failure state



$$\Delta\sigma_1 = \sigma_{1f} - K_0 p_0^{'} - u_0 , \qquad \Delta\sigma_3 = \sigma_{3f} - p_0^{'} - u_0 , \qquad \frac{s_u}{p_0^{'}} = \frac{[1 - A(1-K_0)]\sin\phi^{'}}{1 - \sin\phi^{'} + 2A\sin\phi^{'}}$$

NC clays (typical values)

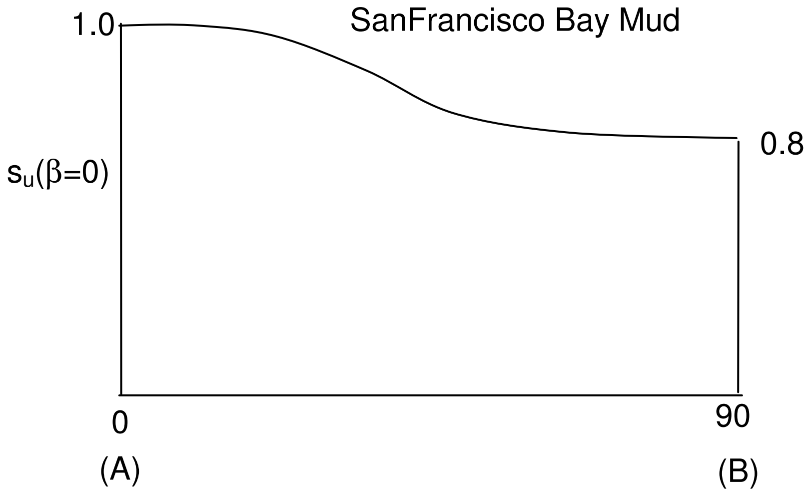
For $\phi^{'}$ =30°

$K_0 = 0.5$

A_f=1 $\Rightarrow \frac{s_u}{p_0^{'}} = 0.17$

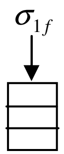
Question : $\phi^{'}_{com} = \phi^{'}_{ext}$ (?), $(A_f)_{compression} = (A_f)_{extension}$ (?)

- Example of undrained strength anisotropy

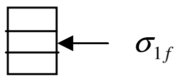


⇒ Combined effect of inherent and stress system anisotropy

A : $\dfrac{s_u}{p_0^{'}} = 0.37$



B : $\dfrac{s_u}{p_0^{'}} = 0.30$



Plane strain tests vs. Triaxial tests.

Plane strain tests comparing to triaxial tests gives;(the effect of intermediate stress σ_2)

① Slight increase (5± %) in s_u.

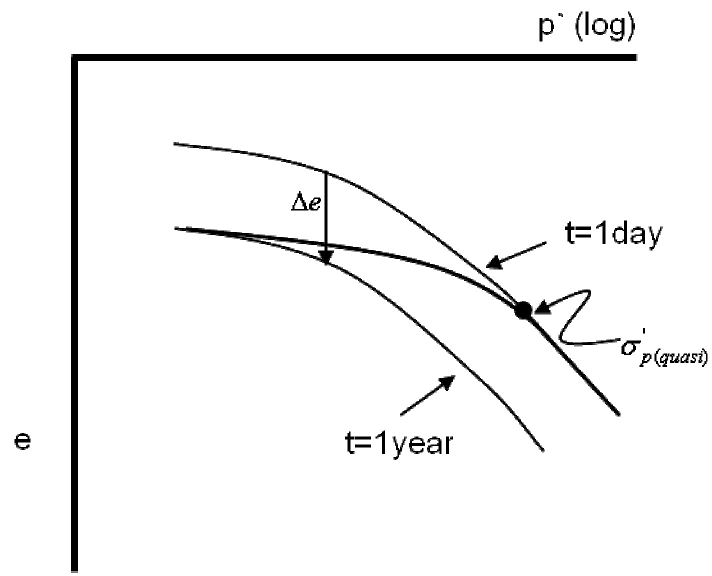
Loading direction	$s_u(TX)/s_u(PL)$
β=0°	0.92±0.5 (5 clays)
β=90°	0.82±0.2 (4 clays)

② Increase ϕ' by 2±2°.

③ Lower strain at failure and increase tendency of the strain softening in plain strain tests, perhaps because of the more general formation of failure plane.

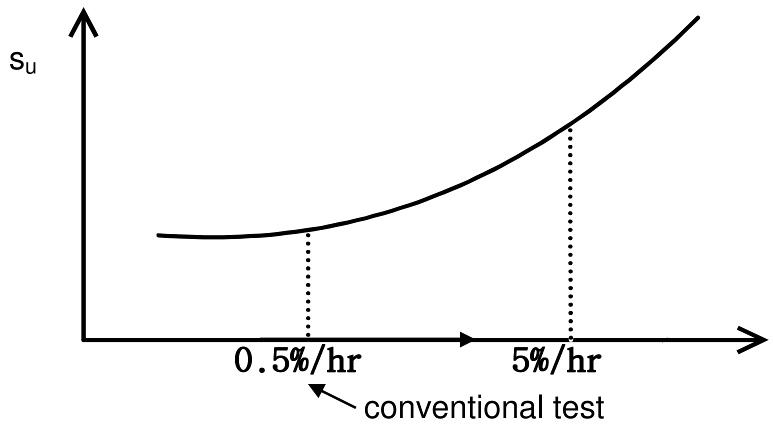
b) Aging effect.

Aging $\rightarrow$ increase undrained strength, owing to decrease of e.



In lab $\rightarrow$ one log cycle of time for secondary compression is required for aging.

c) Rate of shearing.



$(s_u)_{\text{conventional test}} = 1.3\ s_{u(0.5\%/hr}$